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Behavior and Design of Partially-Encased Composite Beam-Columns

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ABSTRACT

Partially-encased composite beam-columns subjected to axial compressive and transverse loads were analyzed using a proposed analytical method, finite element modeling and international approaches, including ACI-318, AISC-LRFD and Eurocode4. In addition, flexural stiffness (EI) was predicted using the recommended formulas in ACI-318 and *EC4*. The results were compared with experimental data to verify of the reliability of these results. The analysis proved that the proposed method could be used for design the partially-encased beam-columns. Research results also indicate that this method gives much close solution to experimental data. The behavior and deformation shape of specimens were close to those found using FEM. The predicted EI from EC4 was reliable in calculating the bending moment using the proposed method while a significant difference in deflection values was pronounced using predicted EI from ACI formula. From calculations, it was found that the ACI-318 approach is accurate and reliable in the design of partially-encased beam-columns.

Keywords: Partially-encased; Composite beam-column; Deformation shape; Finite element modeling; Flexural stiffness.

سلوكية و تصميم الاعمدة-الجسور الخرسانية المركبة والمغلفة جزئيا

الخلاصة

تم في هذا البحث تحليل نماذج (عمود-جسر) من الخرسانة المسلحة و المغلفة جزئيا بمقطع حديدي معرضة الى حمل محوري واخر جانبي باستخدام طريقة مقترحة وطريقة العناصر المحددة فضلا عن ثلاثة طرق تصميم عالمية هي ACI-318 و AISC-LRFD و Eurocode 4. تم حساب صلابة الانحناء (EI) باعتماد طريقتين متضمنتين في ACI و EC4 . اظهرت الطريقة المقترحة نتائج مقبولة جدا عندما تكون صلابة الانحناء (EI) محسوبة من الطريقة المقترحة في EC4 و يمكن اعتمادها في تصميم هذا النوع من الاعمدة-الجسور. كما اظهرت النتائج ان موديل العناصر المحددة اعطى نتائج مطابقة الى حد كبير مع النتائج العملية. اظهرت النتائج ان طريقة تصميم المقترحة في المواصفات الامريكية ACI للأعمدة المأخوذة في هذه الدراسة متطابقة الى حد كبير مع النتائج العملية.

الكلمات الدالة: مغطى جزئيا، عمود-جسر مركب، شكل التشوه، نمذجة عناصر محددة، صلابة انحناء.

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Nomenclatures

δ The maximum deflection.
λ : the relative slenderness.
A_c: Area of concrete.
A_r: area of steel profile.
A_s: area of steel reinforcement.
C_m: a coefficient based on elastic first-order analysis.
COV: the coefficient of variance.
E_c: elasticity modulus of concrete.
E_{cm}: secant modulus of elasticity of concrete.
EI: the flexural stiffness.
E_m: modulus of elasticity.
E_s: elasticity modulus of steel.
f'_c: compressive strength of concrete.
F_{my}: modified yield stress.
F_y: tensile strength of steel reinforcement.
F_{yr}: tensile strength of steel profile.
I_g: gross moment of inertia.
I_s: the steel moment of inertia.
KL: the effective length of column.
L: the length of the specimen.
M_2: Factored moment.
M_c: the magnification moment.
M_{int}: the internal moment.
M_{max}: the maximum moment.
M_{Sd}: the design moment.
P: the axial load.
P_c: critical load of column.
P_e : Euler load.
P_n: nominal axial capacity.
P_o: pure axial capacity.
Q: the lateral load.
r_m: radius of gyration.
SD: the standard deviation.
y_o: the deflection that would exist (if the axial force were absent).
α: the level of significance difference

Introduction

Partially encased steel profiles are one type of composite members used in composite structures. The partially encased steel composite member consists of structural steel section placed in a reinforced concrete between the flanges. The structural steel is rolled or built-up shape. Deriving benefits from combining the structural steel and reinforced concrete, the composite members possess great load-carrying capacity and stiffness owing to composite action. Further, the two edges concrete encasement can partially serve for fire protection, the use of conventional steel connections to the flanges as well as the reduction or omission of formwork. Therefore, the use of the composite members in multi-storey buildings has been proved more popular in recent years.

Many studies have evaluated the members' resistances according to the international codes and suggested which code was closer to the experimental data that have been obtained from previous studies. Mirza and Lacroix[1] have determined comparisons of strengths from 150 physical tests of rectangular composite steel–concrete columns available in the published literature with the strengths calculated from selected computational procedures are conducted. The computational procedures compared in their study included ACI 318, AISC-LRFD and Eurocode4. They recommended that the ACI 318 procedure is closer to experimental data.

ACI 318 and AISC-LRFD were used by Weng and Yen[2] to investigate the difference between these two approaches and to evaluate the accuracy of their strength predictions by comparing to 78 physical test results of concrete-encased composite columns done by previous researchers. It indicated that the ACI-318 approach generally gives closer predictions than the AISC-LRFD approach.

The ACI-318 permits the use of a moment magnifier approach for the design of slender composite columns. This approach is strongly influenced by the effective flexural stiffness (*EI*), which varies due to the nonlinearity of the concrete stress-strain curve and the cracking along the column length among other factors. Tikka and Mirza[3] determined the influence of a full range of variables on EI used for the design of slender, tied, composite columns in which steel shapes were encased in concrete, and also examined the existing ACI EI equations. Approximately 12,000 isolated square composite columns, each with a different combination of specified properties of variables, were simulated and used to generate the stiffness data. The columns studied were subjected to short-term ultimate loads and equal opposite end moments causing symmetrical single curvature bending about the major axis of the encased steel section. They suggested a new nonlinear equation for EI in design of slender composite columns subjected to major axis bending and was proposed as an alternative to the existing ACI *EI* equations.

However, this study is an attempt to satisfy the following objectives:

1. Proposing analytical method to predict the deflections and moment capacities of partially-encased beam-columns sited in Elghazouli and Treadway[4].
2. Testing the accuracy of flexural stiffness formulas recommended by Eurocode4 (EC4) and ACI 318-08.
3. Developing FEM to predict the deformation shape which will be compared with that found using the proposed method.
4. Using three international approaches including ACI-318[7], AISC-LRFD[8] and EC4[9] to predict the load resistance of the same beam-column specimens.
5. Using a statistical analysis approach to examine which approach gives close experimental-to-predicted results.

ACI 318[7], AISC-LRFD[8] and EC4[9] are used to determine the resistances of cross-sections for the beam-columns that have been applied extreme lateral loading, with or without co-existing axial loads representing gravity conditions as shown in Fig. 1, and compare the results with the experimental data sited in Elghazouli and Treadway[4].

Utilized Methods

Proposed Method

The beam deflection and column stability are the main problems concern with beam-column structural members. These problems are caused by in-span transverse loadings and the axial forces (at certain critical values). The axial force acts enlarge the lateral deflection caused by the bending effect to produce additional lateral deflection and moment in the member. These moment and lateral deflection are called secondary moment and deflection[5].

The analysis of a beam-column is more complicated than a beam or a column, closed-form solution of the present beam-columns are available so long as they stay within the elastic behavior in which the moment can be related to the curvature by a linear relationship.

Approximate method was suggested by Chen[6] for beam-columns subjected to end moments but in this paper is proposed to use the assumption of this method to predict the inelastic behavior and maximum moment of the beam-columns adopted herein that assuming the deflected shape of the member resembles a half-sine wave, so the deflection equation will be

$$y = \delta \sin \frac{\pi x}{L} \quad \text{..(1)}$$

from which obtain

$$y' = \delta \frac{\pi}{L} \cos \frac{\pi x}{L} \quad \text{..(2)}$$

and

$$y'' = -\delta\left(\frac{\pi}{L}\right)^2 \sin\frac{\pi x}{L} \quad \text{………..…………..(3)}$$

from the free body diagram shown in Fig. 2(b.) an equilibrium equation can be written

$$Py + \frac{Q}{2}x = M_{\text{int}} \quad \text{…………………………..….(4)}$$

but

$$M_{\text{int}} = -EI \times y'' \quad \text{……………………........….(5)}$$

so equation (4) can be written as

$$P\delta \sin\frac{\pi x}{L} + \frac{Qx}{2} = EI\left(\frac{\pi}{L}\right)^2 \delta \sin\frac{\pi x}{L} \quad \text{……(6)}$$

or

$$\delta(P - P_e)\sin\frac{\pi x}{L} = \frac{-Qx}{2} \quad \text{…………………..(7)}$$

where

$$P_e = (\pi/L)^2 EI \quad \text{…………………..…...…...(8)}$$

Pe is Euler load which constitutes an important reference load in the buckling and stability analysis of structural members. The maximum deflection (δmax) occurs at (*x=L/2*)

$$\delta_{\max} = \frac{QL}{4(P_e - P)} \quad \text{……………………….....…..(9)}$$

then substitute the value of (*δ*) in equation (1) and take the first and second derivative to get the curvature equation

$$y = \frac{QL}{4(P_e - P)}\sin\frac{\pi x}{L} \quad \text{………………....…..(10)}$$

$$y'' = \frac{-QL}{4(P_e - P)}\left(\frac{\pi}{L}\right)^2 \sin\frac{\pi x}{L} \quad \text{………....….(11)}$$

using

$$M = EIy'' \quad \text{……………………………...….(12)}$$

and substituting (*x=L/2*) it can be yielded a maximum moment value

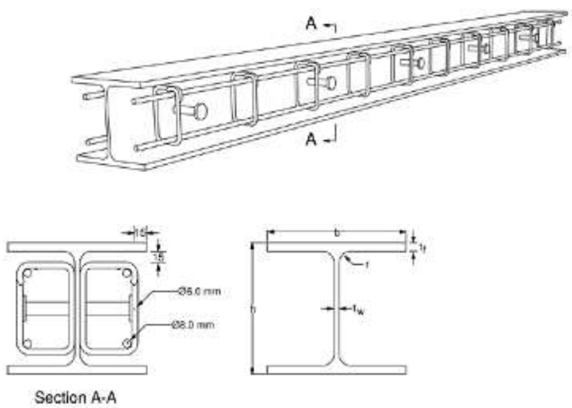
$$M_{\max} = \frac{QLP_e}{4(P_e - P)} = \delta \times P_e \quad \text{………….....…(13)}$$

Finite Element Modeling (FEM)

The software ABAQUS[11] was used to develop a nonlinear three-dimensional (3D) FEM in simulating the partially-encased composite beam-columns. The types of elements were carefully chosen to simulate the parts of the adopted specimens with accurate model. The steel profile was simulated using the four-node isoparametric thin shell element with reduced integration (S4R). A three-dimensional isoparametric solid element (C3D8) was used to simulate the concrete. Three-dimensional truss element (T3D2) was utilized in simulation of the reinforcing bars. The interaction between reinforcement and concrete was modeled using embedded element. The tie elements were used to represent the contact between the thin-walled steel plates and the concrete. Different mesh sizes were considered to select the reasonable mesh that provides accurate results with lesser computational time. It was found that a mesh size about 30 mm is the appropriate one.

The boundary conditions and applying loads of analysis were modeled to follow the testing procedure performed by Elghazouli and Treadway[4] as shown in Fig. 1. The load was applied incrementally using the General Static method available in ABAQUS. Monotonic uniform axial load similar to that in the tests was applied using the displacement control at nodes of the end. The time increment of load did not exceed 5% of the whole displacement.

In the present model, the idealized elastic to perfectly plastic stress–strain relationship is used for utilized steel (i.e. reinforcement and profile). The strain hardening of steel is considered as 1%. The concrete was modeled using damaged plasticity model available in ABAQUS software.



Section	h*	b*	t_f*	t_w*	r*
HEA140	133	140	8.5	5.5	12
HEA200	190	200	10.0	6.5	18
HEAA240	224	240	9.0	6.5	21
* dimension in (mm)					

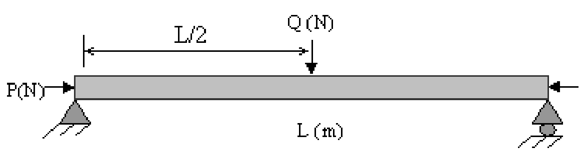


Fig. 1.Details of partially-encased composite profile and applying loads

International Approaches

The most famous specific regulations for the design of partially-encased composite columns in the United States and Europe are included in three different sets of structural design specifications. One is the Building Code for Structural Concrete of the American Concrete Institute (ACI-318), the second is the specification of Load and Resistance Factor Design (LRFD) published by American Institute of Steel Construction (AISC) and the third is the Eurocode4 (EC4). The ACI-318 for the design of the partially-encased composite columns follows the same procedure as that for the reinforced concrete columns. In contrast, the AISC-LRFD is based on analogous to the steel column design. In the EC4 the resistance can be calculated from the cross section and column axial force–bending moment strength interaction diagrams. All ACI, AISC and EC4 design provisions can be applied to concrete-encased structural steel columns and to concrete-filled pipes or tubing.

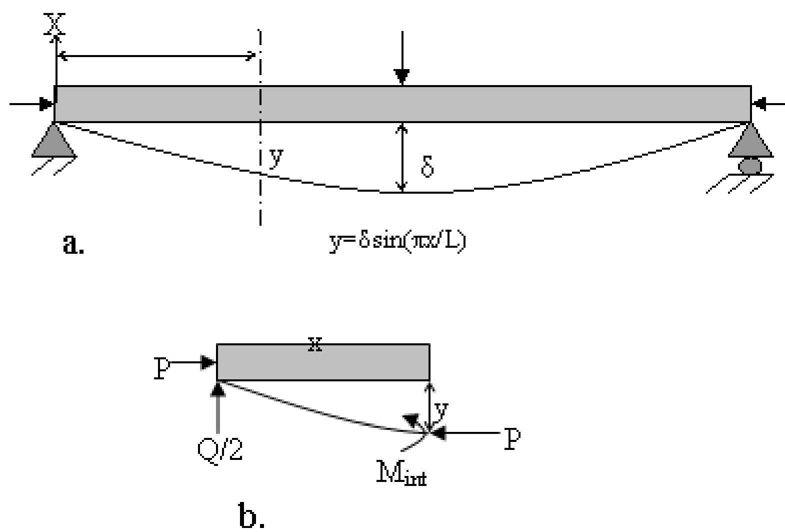


Fig. 2. Proposed method assumptions (a.) half-sine wave along the span (b.) free body diagram of cutting span

ACI 318 Approach

Chapter 10 of the ACI-318[7] building code introduces the concerned strength provisions for all kinds of composite columns. This approach requires that all columns be designed as beam-columns transferring both shear and bending moment at joints. Under uniaxial compression, the nominal compressive strength, of a concrete-encased composite column can be found by summing up the axial-load capacities of the materials that make up the cross section

$P_n = 0.8P_o$,

$$P_o = 0.85 f_c' A_c + F_{yr} A_r + F_y A_s \text{ ....(14)}$$

The columns should be designed according to the factored forces and moments from a second-order analysis. As an alternative to the second-order analysis, design can be based on first-order elastic analysis and moment magnification approach. The magnification moment is expressed as

$$M_c = \delta_{ns} M_2 \text{ ....(15)}$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \geq 1.0 \text{ ....(16)}$$

C_m =1.0 for members with transverse loads between supports, M_2 is Factored moment and P_c is critical load of column, taken as $P_c = \pi^2 EI/(KL)^2$. To account for the variations in stiffness due to cracking, creep and nonlinearity of concrete, the *EI* value of above equation can be conservatively taken as ($0.2E_c I_g + E_s I_s$). The deflection is summation of the imperfection and (y_o) which is the deflection that would exist (if the axial force were absent).

AISC-LRFD approach

Chapter I of the *AISC-LRFD*[8] specification introduces the concerned strength provisions for encased composite columns. In this approach a modified yield stress *F*my, modulus of elasticity *E*m and radius of gyration *r*m were incorporated into steel column design equations for the design of composite columns. This procedure was presented by the Task Group 20 of the Structural Stability Research Council (SSRC) in 1979[4].

The capacity of a partially-encased column is determined from the same equations as that for bare steel columns except the formulas being entered with modified properties F_{my}, E_m and r_m.

For columns designed on the basis of elastic analysis, the factored moment M_u should be determined by a second-order analysis or by the moment magnification method. The moment and deflection magnifier B_1 is expressed as

$$B_1 = \frac{C_m}{1 - \frac{P_u \lambda_c^2}{A_s F_{my}}} \geq 1 \text{ ....(17)}$$

Where C_m is a coefficient based on elastic first-order analysis assuming no lateral translation, for compression members subjected to transverse loading between their supports, C_m=0.85 for members whose ends are restrained and C_m=1.0 for members whose ends are unrestrained.

For a partially-encased composite column symmetrical about the plane of bending, the interaction of compressive and flexural loads should be limited by the following bilinear relationship:

$$\frac{P_u}{\phi_c P_n} + \frac{8M_u}{9\phi_b M_n} \leq 1.0 \text{ for } P_u \geq 0.2\phi_c P_n \text{ ..(18)}$$

And

$$\frac{P_u}{2\phi_c P_n} + \frac{M_u}{\phi_b M_n} \leq 1.0 \text{ for } P_u < 0.2\phi_c P_n \text{ ..(19)}$$

Where ϕ_c = 0.85 and ϕ_b = 0.9.

Eurocode4 Approach

The *Eurocode4* unfactored axial load strengths (or bending moment strengths in cases of pure bending) should be computed from the cross section and column axial force–bending moment strength interaction diagrams. The strength interaction diagrams are generated for partially-encased column and are based on the assumptions and requirements of *Eurocode4* (CEN 1994). *Eurocode4* provides a general method and a simplified method for design of composite columns. The simplified method used and summarized in this study.

The cross-sectional resistance of fully or partially concrete-encased steel sections column to axial compression is the aggregate of the plastic compression resistances of each of its constituent elements as follows:

$$P_{pl.Rd} = A_r f_{yr} + A_c \times 0.85 f_c + A_s f_s \quad .....(20)$$

The elastic critical load *Pcr* of an encased column is calculated using the usual Euler buckling equation and the relative slenderness λ of an encased column in the plane of bending considered is given by

$$\lambda = \sqrt{\frac{P_{pl.Rk}}{P_{cr}}} \quad ....(21)$$

the design axial loading *PSd* satisfies the inequality:

$$P_{Sd} \le \chi P_{pl.Rd} \quad ....(22)$$

in which the value of χ, the strength reduction factor in the plane of buckling considered, is a function of the relative slenderness λ and the appropriate European buckling curve, it is possible to calculate the value of the strength reduction factor χ using:

$$\chi = \frac{1}{\phi + [\phi^2 - \lambda^2]^{1/2}} \le 1 \quad ....(23)$$

in which

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] \quad ....(24)$$

where α is a generalized imperfection parameter which allows for the unfavorable effects of initial out-of-straightness and residual stresses.

Second-order effects on the behavior of an isolated column forming part of a non-sway frame can be taken into consideration approximately by applying an amplification factor *k* to the maximum first-order bending moment *MSd*. The factor *k* is given by:

$$k = \frac{\beta}{1 - \frac{P_{Sd}}{P_{cr}}} \ge 1.0 \quad ....(25)$$

When axial loading and end-moments are both present, β should never be taken as less than 1.0 unless it is calculated by a more exact method.

The design moment *MSd* is the maximum moment occurring within the length of the column, including any enhancement caused by the column imperfections and amplification of the total first-order moments due to the

$$M_{Sd} \le 0.9 \mu_d M_{pl.Rd} \quad ....(26)$$

Statistical Analysis

Statistical analysis including standard deviation (SD), coefficient of variance (COV) and significant (2-tailed, p) was used to verified that a proposed method and others approaches which has enough accuracy for determination lateral deflection and moment properties of beam-columns samples. Table 2 and 3 summarize the analysis results of samples performed at 5% significant level. The analysis verified that the suggested method could be used for determination of lateral deflection and moment properties of beam-columns.

Results and Discussions

Effect of Flexural Stiffness (EI) on Proposed Method

In the calculations of the proposed analytical method and others approaches the most important parameter is the flexural stiffness (EI), particularly in predicting of Euler force (Pe) that reasonably approximates the variations in stiffness due to cracking, creep, and nonlinearity of the concrete stress-strain curve, so some of specific regulations recommended to use equations to predict it. From these regulations are ACI code and EC4.

Table 1: Descriptions of materials, profiles and loads

Specimen	Steel Profile	Plane of Bending	*fy*(MPa)	$P_{Exp.}$ (kN)	$Q_{Exp.}$ (kN)
C14Z2	HEA140	Z-Z	527	470	72
C14Y2B	HEA140	Y-Y	480	470	135
C14Y2A	HEA140	Y-Y	527	470	156
C14Y0	HEA140	Y-Y	527	0	160
C20Z2	HEA200	Z-Z	500	800	203
C20Y2	HEA200	Y-Y	500	800	397
C20Y0	HEA200	Y-Y	500	0	374
C24Y1	HEAA240	Y-Y	520	500	553
C24Y0	HEAA240	Y-Y	493	0	450
C24Y2	HEAA240	Y-Y	493	800	515
f_{cu}=44 MPa, f_c'=35.2 MPa					
L= 2.44 m					

nomenclature of specimens: for C14Y2B, C means column, 14 means the height of the steel profileY is the plane of the bending, 2 means the axial load is about 20% of the critical force.

HEA is the name of steel-section; f_y is yield strength of the steel profile (MPa) ; f_{cu} is the cubic compressive strength of concrete (MPa); f_c' is the cylindrical compressive strength of concrete (MPa); L is the length of the beam-column; P_{Exp} is the axial compressive load (kN) and Q_{Exp} is the transverse load (kN).

ACI code recommended two equations to predict EI in short-term conditions (i.e. neglecting creep effect), they are

$$EI = 0.2E_c I_g + E_s I_s \text{ ……………..……}(27)$$

$$EI = 0.4E_c I_g \text{ …………………………...}(28)$$

Where I_g and I_s are the second moment of are of gross-section and steel profile, respectively, E_s and E_c are the modulus of elasticity of steel and concrete, respectively. E_c can be calculated using $E_s = 4733\sqrt{f_c'}$ (MPa). Equation (27) was derived for small eccentricity ratios and high levels of axial load where slenderness effects are most pronounced. Equation (28) is a simplified approximation to equation (27) and is less accurate.

Eurocode4 recommended an equation to predict the effective flexural stiffness $(EI)_{eff}$ when was needed to determine the internal forces, this equation is

$$(EI)_{eff} = K_o(E_a I_a + E_s I_s + K_e E_{cm} I_c) \text{ ...}(29)$$

Where K_e is a correction factor which should be taken as 0.5; K_o is a calibration factor which should be taken as 0.9; I_a , I_s and I_c is the second moment of areas of steel profile, longitudinal reinforcement and concrete, respectively; E_a and E_s is the elasticity modulus of steel profile and longitudinal reinforcement, respectively and E_{cm} secant modulus of elasticity of concrete which can be calculated from (EC2-1992).

$$E_{cm} = 9500 \times \sqrt[3]{\left(8 + f_c'\right)} \text{ MPa……..…..}(30)$$

It was proposed to use the equations (27 and 29) to predict the *EI* for beam-columns adopted in this study. After predicting moment and deflection values for specimens and analyzed the results statistically by calculating the mean value of the rate of calculated to experimental moment and deflection, standard deviation, coefficient of variation and t-test. It was found that the proposed analytical method gives closed results to predict the moments and deflection when equation (29) was used, since the mean value of the calculated-to-experimental moment was 1.02 with (SD=0.036), (COV=3.5%) and from t-test analysis found that there is no significantly different at level α=0.05 and test mean μo=1, but when equation (27) used to predict (EI) the calculations of moments were closed and the calculations of deflections were not closed, since the mean value of calculated-to-experimental moment was 1.03 with (SD=0.04598), (COV=4.46%) and t-test proved that no significantly different, and the mean value of calculated-to-experimental deflection was 1.14 with (SD=0.1391), (COV=12.2%) and t-test proved that a significance difference at level α=0.05 and test mean μo=1.

From the above it can be indicated that the equation (29) is more accurate to present the proportions of the cross-section elements specially the concrete because it's contribution in the equation is a half of second moment area of the net concrete area and the short-term load secant modulus of elasticity for concrete have taken a consideration the characteristics of cracked composite sections. While the equation (27) recommended by ACI to predict the flexural stiffness showed a little accurate with proposed method, this inaccuracy is the consequences of ACI equation (27) using a constant value of the coefficient 0.2 assigned to E_cI_g as well as ignoring the contribution of longitudinal steel bars to the effective flexural stiffness, it is evident from difference shown in Table 2, Fig. 3 and Fig. 4 that there appears to recommend using equation (29) recommended by Eurocode4 with proposed method.

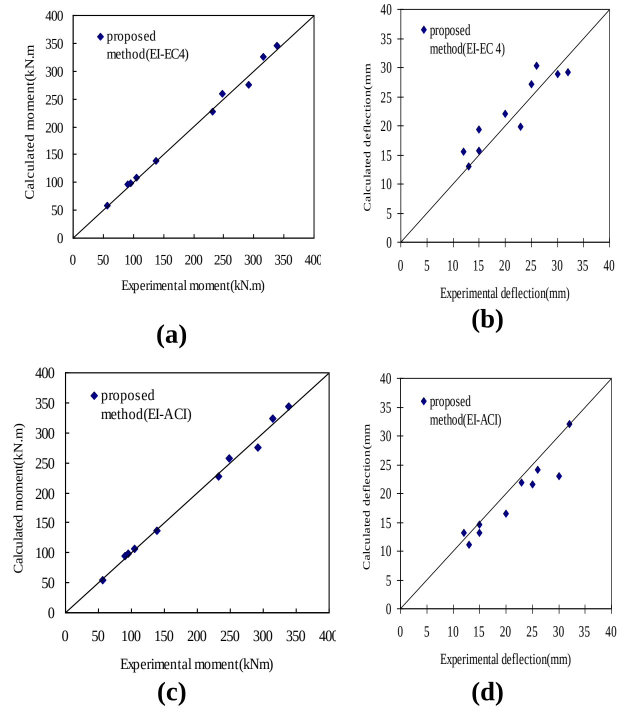


Fig. 3.Comparisons between experimental and predicted moments and deflection for proposed method

The experimental data have been given at yield stage; evaluating the yield stage of the member was required for assessing the effective stiffness, capacity and ductility of a member. In reinforced concrete members, it is customary to consider this point as that corresponding to first yielding of the reinforcement bars. On the other hand, in steel sections, first yield is not normally accompanied by significant increase in curvature. An alternative approach is to consider the effective yield stage, relating to the initiation of a plastic hinge in the member, as that corresponding to reaching yield within the two extreme steel fibers of the section.

Finite Element Modeling Results

A comparison between the experimental and analysis results was adopted to verify the developed finite element model. The comparison of failure mode is shown in Fig. 4 and seemed to be closed. The ultimate lateral loads obtained from the tests and the FEM, as well as the load-deflection curves were reported in Fig.4 and Fig.5, respectively. As indicated in Table (2), a good correlation is achieved between both results for adopted columns. Fig. 6 shows the failure mode of steel profile and seems to be acceptable. From these Figures, it can be indicated the rapprochement between the experimental and FEM results.

Table 2: Calculated to Experimental Ratios Analysis for Proposed method

Specimen	Proposed Analytical Method							
	EI:Eurocode4		EI:ACI 318		EI:Eurocode4		EI:ACI 318	
	M_{cal}	Δ_{cal}	M_{cal}	Δ_{cal}	M_{cal}/M_{exp}	$\Delta_{cal}/\Delta_{exp}$	M_{cal}/M_{exp}	$\Delta_{cal}/\Delta_{exp}$
C14Z2	62.8	32.0	67.7	49.0	1.03	0.913	1.11	1.40
C14Y2B	116.6	28.8	115.5	31.2	1.07	0.961	1.06	1.04
C14Y2A	117.5	30.4	117.5	29.4	1.04	1.17	1.04	1.13
C14Y0	118.5	140.4	118.5	136.5	1.03	1.08	1.03	1.05
C20Z2	144.4	20.6	150.2	30.5	1.01	0.859	1.05	1.27
C20Y2	285.6	22.0	282.9	20.2	1.05	1.10	1.04	1.01
C20Y0	283.1	148.4	283.1	127.7	0.983	1.29	0.983	1.11
C24Y1	352.9	15.8	352.9	16.1	1.02	1.05	1.02	1.07
C24Y0	303.6	60.6	303.6	61.8	0.94	1.01	0.940	1.03
C24Y2	327.6	15.6	327.6	16.0	1.04	1.30	1.04	1.33
Mean					1.02	1.0733	1.03	1.14
SD					0.036	0.148	0.046	0.1391
COV%					3.51	13.8	4.46	12.2
T-test(calculated)					1.8256	1.5658	2.1895	3.2803
p-Probability (that R-square is zero)					0.1012	0.1518	0.0563	0.0095
α(significance level)					0.05	0.05	0.05	0.05
					Not significantly different	Not significantly different	Not significantly different	significantly different

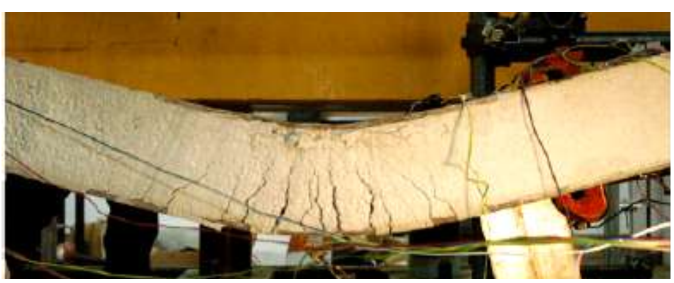


Fig. 4. Verification of deformation shape (specimen C14Y0)

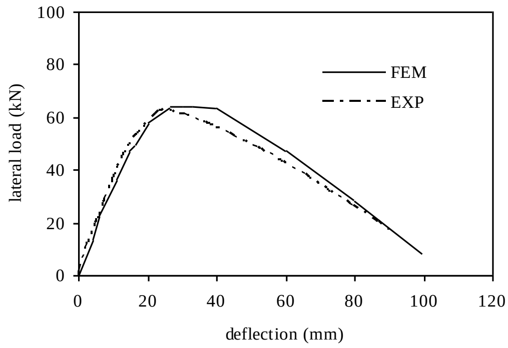


Fig. 5. Comparison of Experimental and FEM load-deflection of specimen C14Z2

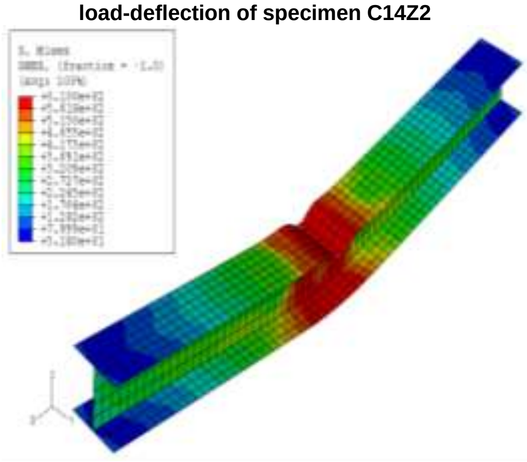


Fig. 6. Deformed profile of specimen C20Y0

International Approaches Differences

Table 3 and Fig. 7 show the comparisons among three international approaches used to predict the moment capacity and deflection of adopted specimens. The table reveals the predicted to experimental moments and deflections ratios and there statistical analysis. The table and figure are evident that the ACI 318 approach is closer than AISC-LRFD and Eurocod4 approaches in predicting of moment and deflection simultaneously but the Eurocode4 is closer than the others in predicting of moment, since for ACI 318-08 the mean value of calculated-to-experimental moment was 1.02 with (SD=0.03714), (COV=3.63%) and from t-test analysis found that there is no significantly different at level α=0.05 and test mean μo=1,and for deflection was 0.981 with (SD=0.1339), (COV=13.6%) and from t-test analysis found that there is no significantly different at level α=0.05 and test mean μo=1. While Eurocode4 has the mean value of the moment capacity-to-experimental moment was 1.01 with (SD=0.032843), (COV=3.26%) and from t-test analysis found that there is no significantly different at level α=0.05 and test mean μo=1.

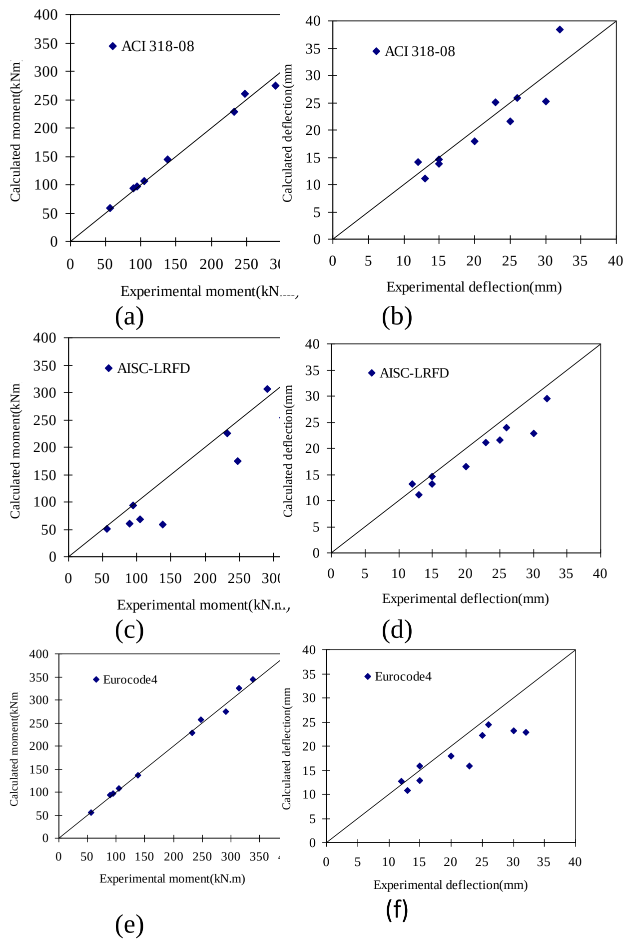


Fig. 7.Comparisons between experimental and predicted moments and deflection for ACI 318, AISC-LRFD and Eurocode4

Regarding the difference of design philosophy adopted in the ACI-318, AISC and EC4 specification, it is noted that the ACI-318 treats the design of concrete partially-encased composite columns through the extension of the design provisions for ordinary reinforced concrete columns. The ACI-318 approach considers the steel shape as an equivalent amount of reinforcement and calculates the capacity of a partially-encased composite column based on a strain compatibility analysis procedure.

On the other hand, the AISC-LRFD approach treats the design of concrete-encased composite columns through the extension of the provisions recommended for bare steel columns. That is, the design of a partially-encased composite column is proceeded by transforming the reinforced concrete portion into an equivalent contribution of steel shape. Then, the composite column is designed using the formulas developed for steel columns. This philosophy may provide a part of the reasons why the AISC-LRFD approach gives less accurate and wider spread predictions as compared with the ten beam-column test results.

Eurocode4 which makes use of the European buckling curves for steel columns, which implicitly take account of imperfections. The simplified method is limited in application to composite columns of bisymmetric cross-section which does not vary with height and based on assumptions that there is full interaction between the steel and concrete sections until failure occurs; geometric imperfections and residual stresses are taken into account in the calculation, although this is usually done by using an equivalent initial out-of-straightness, or member imperfection and plane sections remain plane whilst the column deforms.

Conclusions

The analytical prediction of moment and deflection of ten partially-encased composite beam-columns was presented in this paper. A proposed analytical method, finite element modeling and three international design approaches were used to predict the moment capacity and lateral deflection; these approaches are ACI 318, AISC-LRFD and Eurocode4. From the yielded results and the statistical analysis, the following findings were made:

1. The proposed method gives satisfactory results of moment and deflection estimation based on the predicted EI from EC4.
2. The finite element results which predicted using the developed model are closed to that experimentally captured.
3. Generally, it was found that the ACI 318 design approach is closed in predicting the moment and the deflection simultaneously.
4. The statistical analysis proved that the Eurocode4 is closed in predicting the moment capacity of members.

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